US Army Corps
of Engineers®
Alaska District

DRAFT Feasibility Report and
Environmental Assessment

Homer Navigation Improvements
Homer, Alaska
Appendix B: Geotechnical

May 2026

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ACRONYMS

ASCE	American Society of Civil Engineers
CIRIA	Construction Industry Research and Information Association
CL	Clay of Low to Medium Plasticity
CPTU	Cone Penetrometer Test with Pore Pressure Measurement
EPGA	Effective Peak Ground Acceleration
FS	Factor of Safety
ksf	Kips per Square Foot
LPT	Large Penetration Test
MDE	Maximum Design Earthquake
MHHW	Mean Higher High Water
MHW	Mean High Water
ML	Silt
MLLW	Mean Lower Low Water
MLW	Mean Low Water
NOAA	National Oceanic and Atmospheric Administration
PED	Pre-Construction Engineering and Design
psf	Pounds per Square Foot
SM	Silty Sand
SP	Poorly graded Sand
SPT	Standard Penetration Test
STA	Station
UFC	Unified Facilities Criteria
USCS	Unified Soil Classification System

APPENDIX B GEOTECHNICAL

B.1 Introduction

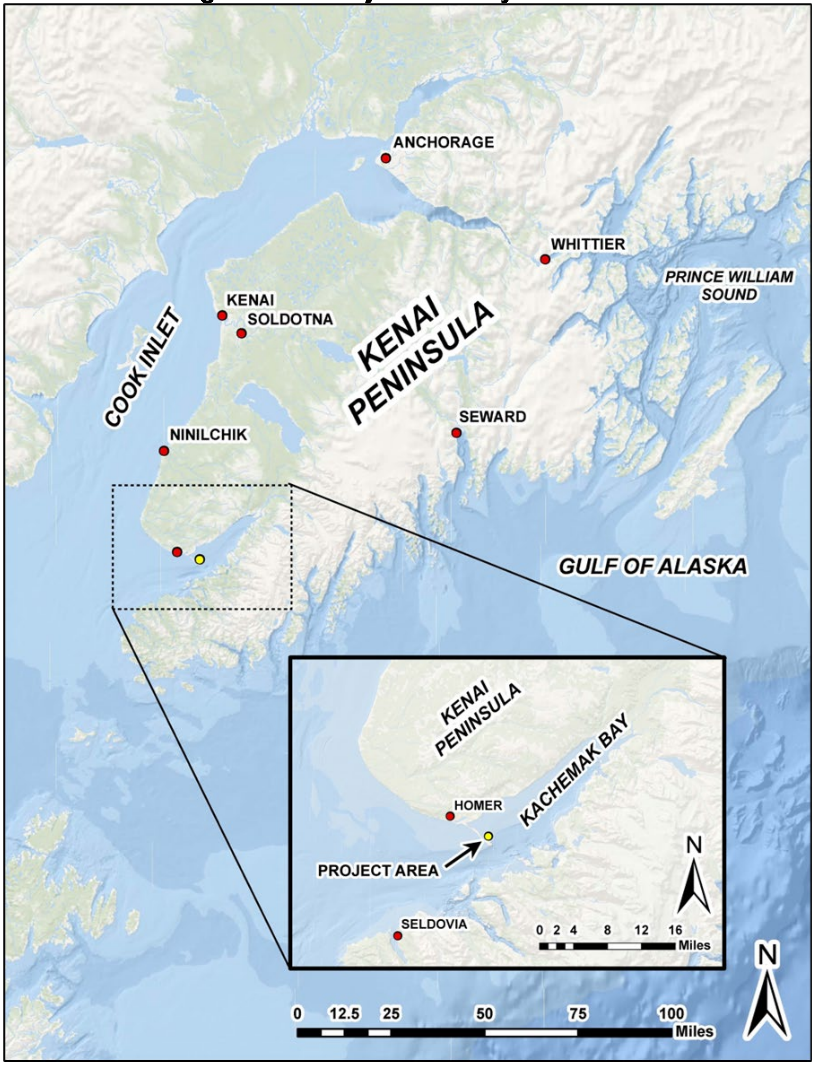
The purpose of this report is to perform a desktop review of geotechnical information, document the anticipated subsurface geotechnical conditions, provide analyses of anticipated site conditions as they pertain to the project described herein, introduce a preliminary geotechnical design, and provide construction criteria for proposed navigation improvements for Homer Harbor in Homer, Alaska. Information and assumptions in this report were developed from a project specific geotechnical investigation and geophysical investigation. The information presented is intended for use by design engineers and planners to evaluate the feasibility of the proposed project. Information in this report is not intended for use in construction contract documents. A more detailed engineering analysis is needed before geotechnical recommendations for the design and construction of the proposed project can be made.

B.2 Location and Project Description

Homer, Alaska, is located on the southwestern edge of the Kenai Peninsula of Alaska and sits on the north shore of Kachemak Bay at the terminus of the Sterling Highway, 227 miles south of Anchorage (Figure B-1). Homer’s population at the time of the 2020 census was 5,522. The city serves as the economic center of the southern Kenai Peninsula and is accessible via air, road, and water year-round. The Homer Harbor is located at the end of the Homer Spit.

Homer Harbor is a regional port serving the needs of commercial vessels operating across southcentral and western Alaska in the maritime industrial, marine transportation, and commercial fishing industries. Demand has outgrown the harbor’s ability to serve this fleet safely and efficiently. Commercial vessels over 200 feet in length cannot access the harbor due to depth limits and the configuration of the harbor entrance. Moorage is often at capacity even after rafted three to four vessels abreast. The harbor is projected to continue to face safety and capacity issues unless action is taken.

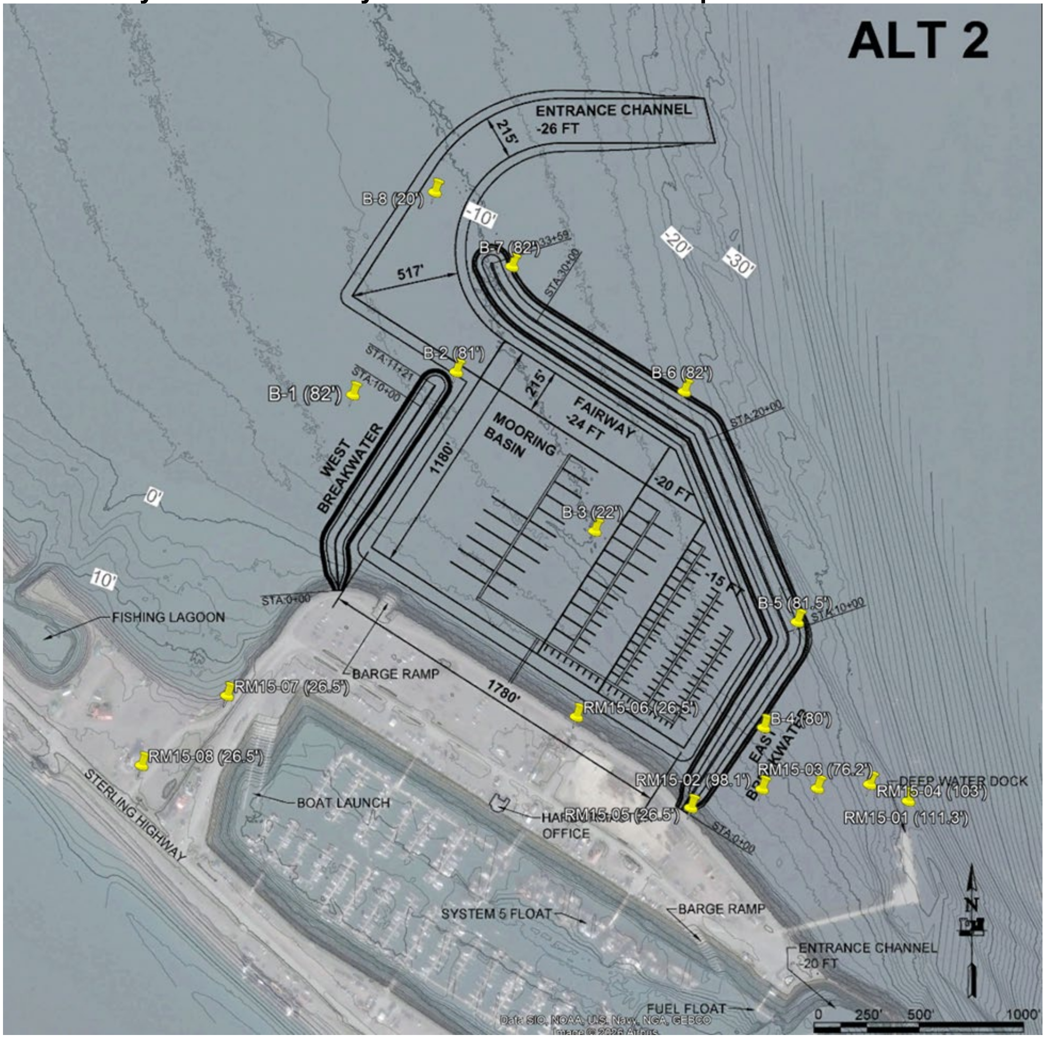
Figure B-1. Project Vicinity of Homer



B.3 Tentatively Selected Plan: Alternative 2

The Tentatively Selected Plan consists of two breakwaters (east and west) with crest heights of +30 feet MLLW enclosing the mooring basin. The basin will be dredged to depths ranging from -20 to -24 feet MLLW, with an entrance channel dredged to -26 feet MLLW. A plan view of Alternative 2 can be found in Figure B-2.

Figure B-2. Layout of Tentatively Selected Plan with Site-Specific and Historic Boreholes



B.4 Geotechnical Investigations

The geotechnical investigation for the Homer Navigation Improvements Feasibility Study (conducted by Shannon & Wilson, Inc. in October 2025) included eight offshore boreholes to depths of 22 to 82 feet below mudline (-7.4 to -98.8ft MLLW). Subsurface soils included a layer of very soft to soft surface marine deposits overlying a very soft to stiff lean clay to sandy lean clay layer, followed by a very soft silt to sandy silt layer that transitions to medium dense consistency with depth. This report can be found in Attachment A.

A geotechnical investigation for the Homer Deep Water Dock Feasibility Study (conducted by R&M Consultants for the City of Homer in July 2015 and published in 2017) included eight boreholes (four onshore, four offshore) to depths of 26.5 to 111.3 feet below mudline (32 to -147.1ft MLLW). Subsurface soils included a layer of loose to medium-dense beach sand overlying very soft to stiff lean clay with sand, followed by firm to stiff sandy silt and loose to medium-dense silty sand. This report can be found in Attachment B.

Information from both of these sites was used to develop the subsurface soil profile and design assumptions.

A geophysical (sub-bottom profiler) survey was conducted within the project footprint by Shannon & Wilson 29 July through 12 August 2024 and 13 to 18 October 2024 as part of the site-specific geotechnical investigation. The results of the sub-bottom profiler survey indicate a zone of weaker return offshore (indicating softer surface sediments such as clay and silt) and a zone of stronger return near the shore (correlating to the presence of sand, gravel or cobbles in surface sediments). The entirety of the east breakwater lies within the zone of softer surface soils while the west breakwater generally lies within the zone of the sand/gravel/cobble surface sediments. An unconsolidated unit of varying thickness (between 0 and 30 feet thick) overlying a consolidated unit was noted throughout the survey area. The geophysical data indicated chaotic deposition of the soil layers. Layer composition was difficult to discern due to the chaotic deposition. Fifty-three subsurface targets (identified as buried rocks or objects) were detected throughout the survey area. This geophysical survey report can be found in Attachment F.

A map of the site-specific and historic boreholes overlayed on the breakwater alignment can be found in Figure B-2.

B.5 Regional Geology

Homer lies within the Cook Inlet–Susitna Lowland, a physiographic region characterized by moderate elevation changes and landforms shaped by stagnant ice and glacial debris from past ice ages (Wahrhaftig 1965). The landscape reflects the influence of significant glacial coverage during the late Pleistocene, as demonstrated by both the terrain and the layering of soils in the area (Coulter et al. 1965). The region typically lacks continuous permafrost (Ferrians 1965).

The subsurface geology near Homer consists largely of Tertiary-age sedimentary rocks that contain coal and are only slightly altered by tectonic activity. These formations are blanketed by deposits from glacial meltwater, ice movement, and ancient lakes. Homer Spit itself developed on an underwater terminal moraine left by the Kachemak Bay glacial lobe. Today, the spit is made up of well-sorted beach sands that rest atop older marine and glaciolacustrine sediments. With its tectonic activity, Homer lies within a high-risk tsunami zone. A discussion of the tsunami hazard can be found in Appendix A (Hydraulics and Hydrology) of the feasibility report.

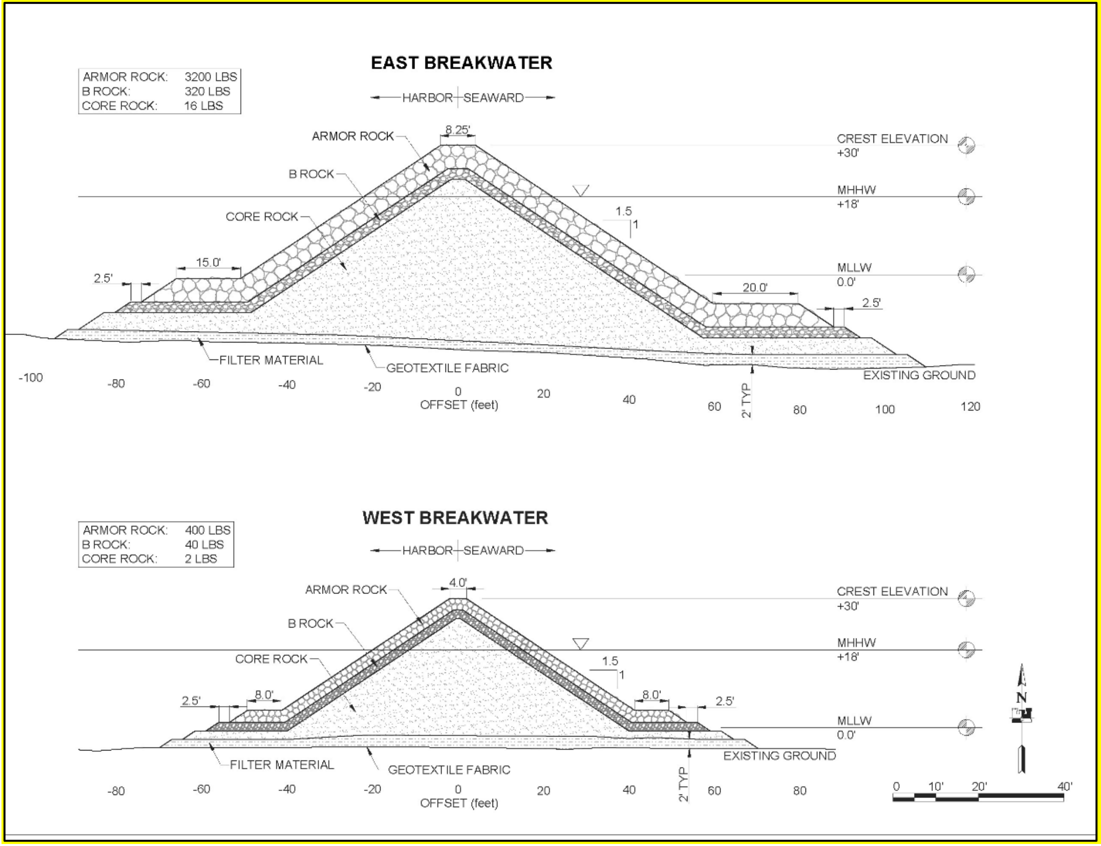
B.6 Geotechnical Design Considerations

It is important that prudent consideration be given to certain subsurface conditions and construction aspects including foundation soils, slope stability, seismicity, and settlement. This engineering analysis was completed using the available data from the site-specific geotechnical investigation and the 2015 Deep Water Dock geotechnical investigation discussed in Section B.4. This engineering analysis, along with all assumptions, will need to be reviewed and updated in the pre-construction engineering and design (PED) phase.

B.6.1 Breakwater Cross-Sections

The preliminary cross-sections for the breakwater are shown in Figure B-3. Each breakwater will be constructed using prefabricated vertical drains (PVDs, also known as wick drains) placed on a 7-foot triangular (the current cost estimate reflects a 7-foot square grid) grid to a depth of 60 feet below mudline within the footprint. The critical section is between STA 10+00 and STA 20+00, where the breakwater is tallest (approximately 47 feet), foundation soils are softer, and the seaside slope drops off.

Figure B-3. Preliminary Breakwater Cross-Sections



B.6.2 Anticipated In Situ Soil Properties

A typical soil profile was developed using the site-specific and historical geotechnical data. All feasibility-level geotechnical designs used this soil profile and given in Table B-1. Typical unit weights were taken from Table 5-2 of *Foundation Design: Principles and Practices* (Coduto 2001). The range of effective internal friction angles were estimated in accordance with Table 3-6 of *Engineering Manual (EM) 1110-1-1905*, *Bearing Capacity of Soils* (2025) and laboratory testing results were used to determine the recommended value for Marine Unit 1.

Table B-1. Typical Soil Profile and Properties

Soil Unit	[1]Depth (ft)	Layer Thickness (ft)	[2]Stiffness/ Consistency	USCS	[3]Wet Unit Weight (pcf)	[3]Friction Angle (degrees)	[3]Cohesion (psf)
Marine Surface Deposits (Silt)	0 to 11	8 to 27	Very Soft to Soft	ML, SM, SP-SM	95 - 135 (100)	27 - 36 (28)	N/A
Marine Unit 1 (Lean Clay)	11 to 57	10 to 46	Very Soft to Stiff	CL, ML	100 - 135 ([4]130 and [5]115)	18 – 35 ([4]18 and [5]0)	100 – 1250 (1000)
Marine Unit 2 (Sandy Silt)	57 to 87	14 to 40	Very Soft to Stiff	ML	100 - 135 (120)	27 -35 (30)	N/A
Marine Unit 3 (Silty Sand)	87 and below	N/A	Loose to Very Dense	SM	115 - 135 (125)	27 - 34 (31)	N/A

[1] Typical soil section used for all analyses herein.
[2] Based on LPT/SPT blow counts from the site-specific and 2015 Deep Water Dock site investigations.
[3] Typical range and recommended design value (in parentheses).
[4] Parameters that are considered “improved” (post-consolidation) and apply to drained analyses only.
[5] Parameters that are considered “unimproved” (pre-consolidation) and apply to undrained analyses only.

Consolidation parameters and assumptions for Marine Unit 1 were derived from five one-dimensional consolidation tests (three from the site-specific investigation and two from the 2015 Deep Water Dock investigation) and five consolidated undrained triaxial compression tests (all from the site-specific investigation). Table B-2 summarizes the consolidation parameters for Marine Unit 1.

The recommended initial void ratio was selected using the (approximate) 33rd percentile of the values from the site-specific one-dimensional consolidation and triaxial tests. This percentile was chosen to be a moderately conservative representative value. Values from the 2015 Deep Water Dock laboratory testing were excluded as they were statistical outliers on the upper end and likely due to sample disturbance before testing.

The recommended compression and recompression indices (C_c and C_r, respectively) were selected using the average of the data set. The average was selected because the values for these tests tended to fall near the lower or higher end of the range with no meaningful correlation to observed soil conditions (such as SPT blow counts) within the project site.

The recommended coefficient of consolidation (C_v) ranged from 10^{-4} to 10^{-6} ft^2/s for the anticipated range of stresses for this project. C_v was reported by the laboratory using the logarithm-of-time and square-root-of-time methods and was reviewed and confirm by USACE-AD. The data predominately lies between 10^{-5} and 10^{-6} ft^2/s for both methods. 10^{-6} ft^2/s was selected as the design value to capture the worst-case scenario and inform construction methodology.

Based on the recommended values shown in parenthesis in Table B-2, Marine Unit 1 is considered highly to very highly compressible according to Table 3-10 of *Engineering Manual (EM) 1110-1-1905*, *Bearing Capacity of Soils* (2025). The recommended design values are generally within their respective typical ranges as given in Table 3-9 of *Engineering Manual (EM) 1110-1-1905*, *Bearing Capacity of Soils* (2025).

Table B-2. Marine Unit 1 Consolidation Parameters

Parameter	[1]Selected Value for Marine Unit 1
C_c	0.26 – 0.73 (0.45)
C_r	0.02 – 0.04 (0.03)
e_0	0.83 – 1.17 (0.94)
H_{dr}	8 – 46 ft (46 ft)
$H_{dr,wick}$	5 ft
C_v	10^{-6} – 10^{-4} ft^2/sec (10^{-6} ft^2/sec)
[1]Typical ranges are given and recommended design values are in parentheses.	

B.6.3 Causeway Engineering Properties

The proposed causeway embankment is composed of several different materials categorized as: Armor Rock, B Rock, C Rock, and Filter Material. An assumed porosity (n) value of 37% (0.37) was used as recommended by EM 1110-2-1100 for in-place large stone products. A conservative specific gravity of 2.65 was used for all rock products for the project, which is a common specific gravity for rock products produced in the region.

Typical internal friction angles for rock fill and armor stone range from 40° to 55° according to Table 5.67 in *The Rock Manual* (CIRIA 2007) and are consistent with literature (Fang 2004). Based on an assumed porosity of 0.37, an internal friction angle of 45° was selected for the Armor, B, and Core rock. The engineering properties of the embankment fill materials are shown in Table B-3.

The dry unit weight of rock was calculated using the following relationship between specific gravity, porosity, and unit weight of seawater:

$$\gamma_d = G_s(1-n)\gamma_w$$

$$\gamma_d = \text{Estimated Dry Unit Weight (pcf)}$$

$$G_s = \text{Relative Density (Specific Gravity)}, (-)$$

$$n = \text{Porosity (assumed 37\%)}$$

$$\gamma_w = \text{Unit Weight of seawater (64.0 pcf)}$$

$$\gamma_d = 2.65 \cdot (1-0.37) \cdot 64.0\text{pcf} = 107 \text{ pcf}$$

This value was used for all rock material used in the design of the breakwater system. Because of the free-draining nature of the rock materials, the dry, wet, and saturated unit weights were assumed to be the same. The Filter Material was assumed to be a well graded gravel with sand with a unit weight representative of the associated friction angle. The unit weight of the filter material will be greater than the other rock materials as it is composed of smaller particle sizes.

Table B-3: Design Embankment Material Properties

Breakwater Unit	Stone Size (lbs)	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Internal Friction Angle (degrees)	Porosity (-)	Specific Gravity (-)
Armor Rock	3200	107	107	45	0.37	2.65
B Rock	320	107	107	45	0.37	2.65
Core Rock	16	107	107	45	0.37	2.65
Filter Material	N/A[1]	130	130	36	0.21	2.65

[1] Filter Material will consist of material larger than #200 sieve and smaller than 3” diameter.

B.6.4 Design Factors of Safety

Appropriate factors of safety must be selected to ensure adequate performance of the project throughout its design life. Three important considerations in determining appropriate factors of safety include: uncertainties in the conditions being analyzed, the consequences of failure, and the acceptable performance. Table B-4 provides applicable factors of safety and source documents, which include procedures for performing the analysis.

Table B-4. Applicable Factors of Safety

Reference	Analysis Condition	Minimum Factor of Safety
EM-1110-1-1905 (2025)	Bearing Capacity	3.0
EM 1110-2-1902 (2003)	Slope Stability, End of Construction	1.3
EM 1110-2-1902 (2003)	Slope Stability, Long Term	1.5
EM 1110-2-1902 (2003)	Slope Stability, Earthquake Loading	1.0
ASCE 61-14	Slope Stability, Post-earthquake Long Term	1.1

B.6.5 Tide Conditions

The tides at Homer are generally mixed semi-diurnal with two highs and two lows occurring daily. Tide levels, referenced to mean lower low water (MLLW), are shown in Table B-5. Water level data is from the Coal Point, AK station in the National Oceanic and Atmospheric Administration (NOAA) online database.

Table B-5. Tidal data for Homer Harbor, Alaska reference to MLLW.

Tide	* Elevation (feet)
Highest Astronomical Tide (observed)	+23.8
Mean Higher High Water (MHHW)	+18.4
Mean High Water (MHW)	+17.6
Mean Tide Level	+9.6
Mean Low Water (MLW)	+1.7
Mean Lower Low Water (MLLW)	0.0
Lowest Astronomical Tide (observed)	-6.2
* Source: NOAA National Ocean Surface	

B.6.6 Seismic Design Parameters

Homer is in seismically active Southcentral Alaska where large magnitude earthquakes occur. Structures must be designed to meet or exceed seismic requirements in *Engineering Regulation (ER) 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects.* Liquefaction settlement was analyzed using the site-specific and 2015 Deep Water Dock geotechnical data.

The proposed structure is assigned a Seismic Design Category D per Section 11.6-1 of American Society of Civil Engineers (ASCE) 7-22, since the mapped spectral response acceleration parameter at 1-second period, S_1, is less than 0.75 and the short-period response acceleration parameter, S_{Ds}, is greater than 0.50 at the project site. Seismic data for Homer, Alaska was determined using the ASCE 7 Hazard Tool and is shown in Table B-6. The specified design ground motions are for Soil Class E and Seismic Design Category D. Seismic design ground motion parameters are provided for ASCE 7-22.

Table B-6. Seismic Design Ground Motion Parameters

Parameter	ASCE 7-22
Seismic Design Category	D
Site-Specific PGA_M	0.42
S_1	0.6
S_{D1}	2.0
S_s	1.5
S_{Ds}	1.0

The proposed facility is assigned a Risk Category II in accordance with Table 2-2 of the UFC 3-301-01 Structural Engineering (2023) since the structure will require tsunami design factors and it does not fit into the other higher risk categories. A summary of these seismic design parameters can be found in Attachment C.

B.7 Preliminary Geotechnical Analysis of TSP

The following sections are based on review of the site-specific geotechnical investigation, geophysical survey, and historical geotechnical reports. These sections are for the feasibility analysis of alternatives only and are not adequate for a formal design.

B.7.1 Bearing Capacity Analysis

The allowable bearing capacity “Q_a” is the ultimate bearing capacity “Q_u” divided by an appropriate factor of safety “FS”. A reasonable factor of safety is based on the available subsurface and surface information, variability of the soil, soil layering and strengths, type and importance of the structure and past experience with like structures. According to EM 1110-1-1905, *Geotechnical Design of Shallow Foundations on Soils* (2025), the minimum factor of safety for marine structures is 3.0.

Meyerhoff’s general bearing capacity equation was used to check the subgrade. For this analysis, and due to its sand content, the in-situ surface soil (the surface marine deposit) is assumed to be in a drained condition. Assumed soil parameters were taken from Table B-1. Equivalent foundation width is assumed to be 175 feet. The soil is assumed to be cohesionless and embedment depth is zero, resulting in two of the bearing capacity factors drop out; N_q and N_c, and the factor for unit weight “N_γ” is 17.1. The unit weight of sea water is assumed to be 64pcf. The ultimate bearing capacity is as follows:

$$Q_u = \frac{1}{2}\gamma' B N_\gamma = \frac{1}{2} \cdot (100pcf - 64pcf) \cdot 190ft \cdot 14.6 = 49.9\ ksf$$

$$B = Breakwater\ foundation\ width\ (ft)$$

$$\gamma' = Buoyant\ unit\ weight\ (psf)$$

Based on the calculated ultimate bearing capacity of the soils at 46.0 ksf, the allowable bearing capacity, Q_a, is shown in the equation below.

$$Q_a = \frac{Q_u}{FS} = \frac{49.9ksf}{3.0} = 16.6\ ksf$$

The loaded area is essentially flat with very little relief; therefore, no eccentric loading is assumed. For the most conservative factor of safety, a completely dry revetment over a completely saturated subgrade was used when calculating the embankment loading. Calculation of the embankment loading is:

$$Q = (3ft \cdot 130pcf) + (46ft \cdot 107pcf) = 5.3\ ksf$$

The equation below shows the estimated factor of safety for the proposed structure:

$$FS_{bearing\ capacity} = \frac{Q_a}{Q} = \frac{16.6}{5.3} = 3.1$$

The bearing capacity analysis indicates the proposed revetment will not exceed the allowable bearing capacity of the surface marine deposits.

However, the bearing capacity of the very soft lean clay layer is of much greater concern. The lean clay is assumed to have 10 feet of overburden (the surface marine deposits layer) in the critical breakwater section. In an undrained condition, the lean clay layer is assumed to have unit weight, internal friction angle, cohesion, and effective foundation width are assumed to be 115 pcf, 0 degrees, 1000 psf, and 185 feet respectively. It is conservative to ignore the surface marine deposit in the bearing capacity analysis because doing so neglects both its shear strength and surcharge effects, resulting in a lower calculated capacity. The effective foundation width B_{eff} was calculated as:

$$B_{eff} = B + z = 190ft + 11\ ft = 201\ ft$$

$$z = Depth\ below\ mudline\ to\ top\ of\ lean\ clay\ layer$$

With the effective foundation width, the surcharge load at depth is equal to 5.0ksf. Assuming that the angle of internal friction is 0 degrees and the foundation is not embedded, the bearing factors N_q and $N\gamma$ drop out. The bearing capacity factor for cohesion "N_c" is 5.7. The ultimate bearing capacity is as follows:

$$Q_u = \frac{2}{3} cN_c = \frac{2}{3} \cdot 1000 \cdot 5.7 = 3.8\ ksf$$

The allowable bearing capacity, Q_a, can then be calculated as shown in the equation below.

$$Q_a = \frac{Q_u}{FS} = \frac{3.8\ ksf}{3.0} = 1.3\ ksf$$

The estimated factor of safety for the proposed structure can then be calculated:

$$FS_{bearing\ capacity} = \frac{Q_a}{Q} = \frac{1.3ksf}{5.0ksf} = 0.26$$

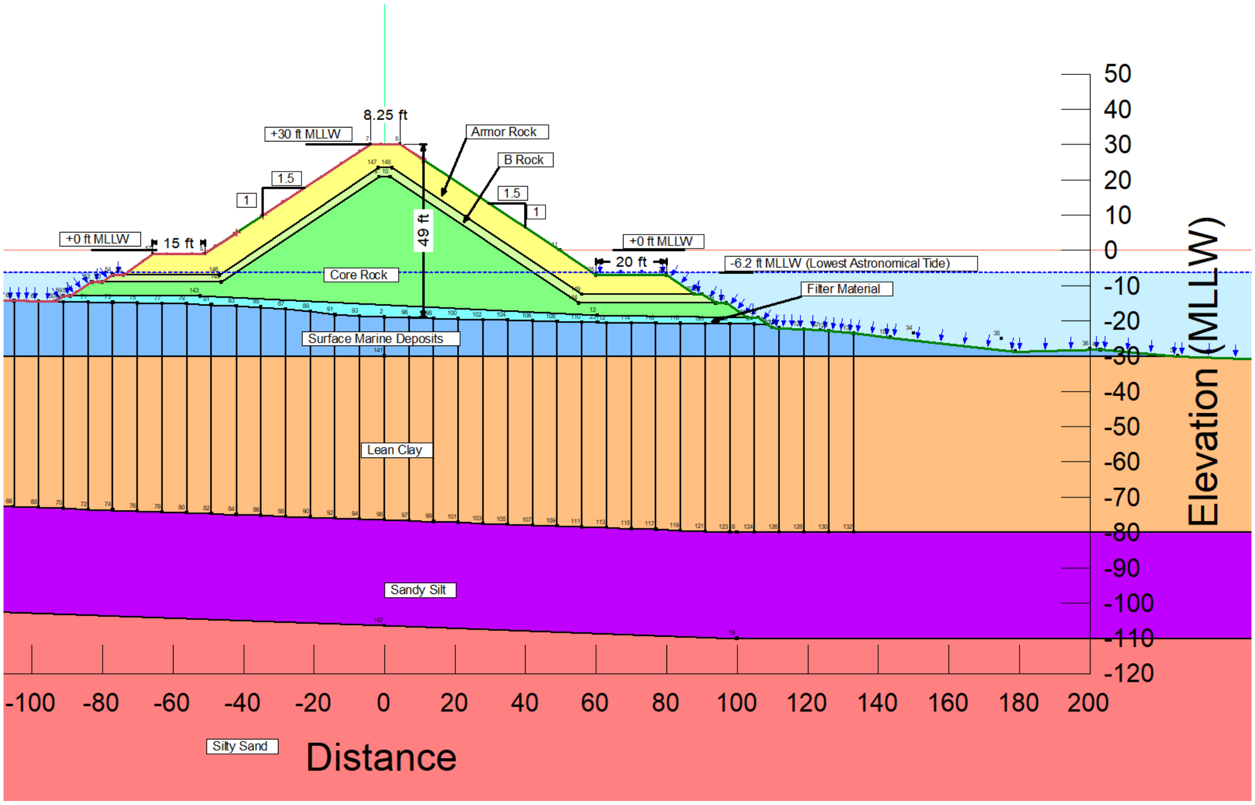
In an undrained state the bearing capacity of the lean clay layer is not sufficient to support the breakwater. With the addition of wick drains the lean clay layer will exhibit drained consolidated behavior as the dissipation rate of excess pore pressures will be accelerated. The drained and consolidated lean clay layer will also have increased strength parameters as shown in Table B-1. With these updated parameters, the estimated factor of safety increases to 3.7.

B.7.2 Slope Stability Analyses

Analytically, there are three different conditions that must be evaluated for slope stability analyses: static, pseudo-static, and post-liquefaction. The critical section is identified as the East Breakwater section in Figure B-4 and was modeled and analyzed in Slope/W®; part of the GeoStudio 2024® software package. The conventional limit-equilibrium methodology (Spencer Method) was used for the primary analysis. This method identified the most critical slip surface with the lowest factor of safety from numerous slip surfaces. To enhance the search process for the critical slip surface and because the slip surface typically is not circular, a Monte-Carlo type optimization technique (a feature of the GeoStudio package) was employed. All slope stability analysis figures can be found in Attachment D.

The analyses presented herein are based on geotechnical data obtained from the site-specific geotechnical investigation and the nearby 2015 Deep Water Dock geotechnical investigation. Material parameters used in the analyses are summarized in Table B-1.

Figure B-4. Modeled Critical Section and In-situ Soil Profile for the East Breakwater



B.7.2.1 Global Slope Stability Analysis

A slope stability analysis was performed to evaluate the performance of the breakwater under varying drainage and loading conditions. Two soil behavior scenarios were modeled: an undrained condition (referred to here as “unimproved”), representing the case without wick drains where excess pore pressures remain in the lean clay layer (only cohesion), and an improved condition, representing the case with wick drains installed, where rapid consolidation allows the clay to gain strength through dissipation of pore pressures (cohesion and internal friction). For each scenario, both oceanside and landside slope failures were analyzed to capture potential sliding mechanisms on either side of the structure. This approach provides a direct comparison of stability outcomes between the no-drain and wick-drain cases, while ensuring that both critical slip surfaces are considered in the assessment. Lowest Astronomical Tide was used as the water table for all long-term analyses and is conservative. The results of these stability analyses can be found in Table B-7.

Table B-7: Factors of Safety for Long Term Steady State

Critical Cross Section Analysis	Tide Condition (ft, MLLW)	Factor of Safety	
		Computed	Minimum
Long Term Steady State Landside (Improved)	-6.2 MLLW	1.6	>= 1.5
Long Term Steady State Landside (Unimproved)	-6.2 MLLW	0.8	>= 1.5
Long Term Steady State Oceanside (Improved)	-6.2 MLLW	1.5	>= 1.5
Long Term Steady State Oceanside (Unimproved)	-6.2 MLLW	0.6	>= 1.5

A post-earthquake long-term stability analysis was conducted to assess the performance of the breakwater following seismic loading. Both the seaward and landward sides were analyzed to evaluate potential failure mechanisms under reduced material strengths. To account for the effects of earthquake shaking, the shear strength parameters of each soil layer were reduced by 20%, reflecting a conservative estimate of strength loss due to seismic loading and associated strain softening. This approach provides an assessment of the structure's residual stability once shaking has ceased and pore pressures have dissipated. The results of these stability analyses can be found in Table B-8.

Table B-8. Factors of Safety for Post-Earthquake Long-Term Stability

Critical Cross Section Analysis	Tide Condition (ft, MLLW)	Factor of Safety	
		Computed	Minimum
Post-Earthquake Landside (Drained)	-6.2 MLLW	1.3	>= 1.1
Post-Earthquake Oceanside (Drained)	-6.2 MLLW	1.4	>= 1.1

B.7.2.2 Seismic Stability Analysis

Slope stability was evaluated during earthquake loading for the critical profile using Slope/W®. A pseudo-static evaluation was performed using the long-term strength parameters of the foundation soils and embankment fill and a pseudo-static loading condition was applied based on the horizontal seismic coefficient (kh). MLW was used as the water table in the seismic analyses as the probability of the design earthquake occurring alongside anything lower than MLW is unlikely and too conservative. Computed factors of safety for the critical cross section and seismic parameters used are shown in Table B-9.

Table B-9. Factors of Safety for Seismic Loading

Critical Cross Section	Earthquake Event	Tide Condition (ft, MLLW)	EPGA (g)	Kh (g)	Factor of Safety	
					Computed	Minimum
Oceanside	MDE	+1.66 MLLW	0.42	0.21	0.7	< 1
Landside	MDE	+1.66 MLLW	0.42	0.21	0.7	< 1

B.7.3 Liquefaction Analysis

When saturated noncohesive soil deposits are subjected to an earthquake, pore water pressures have potential to increase due to dilation and consolidation of the soil grain structure. Excess pore pressures can lead to a loss of shear strength or liquefaction. As the pore pressures dissipate towards the surface, the soil structure may undergo some volume change which can manifest as settlement or lateral displacement. Soil conditions and laboratory data obtained from the site-specific and 2015 Deep Water Dock site investigations were used to analyze liquefaction potential.

Idriss and Boulanger (2008) was used to assess the liquefaction potential of the site in its current condition. The analysis was based on the simplified method using corrected blow count data collected during drilling. Any uncorrected LPT field blow counts were adjusted and normalized to a Standard Penetration Test (SPT) blow count.

The seismic data used for the soil liquefaction analysis is provided by the ASCE 7-22 Seismic Hazard Tool and shown in Table B-6. Based on the assumed subsurface characteristics and the seismic hazard for the site, liquefaction potential was analyzed for all boreholes. The minimum allowable factor of safety for liquefaction potential, per EM 1100-1-1905 (2025), is 1.0. There were several thin zones of soil that produced a factor of safety less than 1.0 with respect to liquefaction potential. Based on the liquefaction analysis, depth of potentially liquefiable soils, and discontinuity of liquefiable layers across the project area, no liquefaction mitigation is required. The low magnitude of seismic-induced settlement and lateral spreading further supports this conclusion. Factors of safety for the historic and project boreholes are shown in Table B-10 and Table B-11 and the Liquefaction Analysis Worksheets are in Attachment E.

Table B-10. Liquefaction Analysis Results for Historic Boreholes

RM15-01				RM15-02				RM15-03				RM15-04			
Depth (ft)			FS	Depth (ft)			FS	Depth (ft)			FS	Depth (ft)			FS
0	to	15	#N/A	0	to	4	0.46	0	to	8	0.20	0	to	35	#N/A
15	to	40	#N/A	4	to	8	>2	8	to	17	#N/A	35	to	43	#N/A
40	to	44	#N/A	8	to	13.5	>2	17	to	25	#N/A	43	to	54	#N/A
44	to	48	#N/A	13.5	to	20.5	0.25	25	to	40	#N/A	54	to	64	#N/A
48	to	54	#N/A	20.5	to	29	#N/A	40	to	45	#N/A	64	to	77	#N/A
54	to	58	#N/A	29	to	35	#N/A	45	to	51	#N/A	77	to	84	#N/A
58	to	61	#N/A	35	to	41	#N/A	51	to	61	#N/A	84	to	93	1.00
61	to	71	#N/A	41	to	47.5	#N/A	61	to	72	#N/A	93	to	99	>2
71	to	80	0.29	47.5	to	55	#N/A	72	to	76.2	#N/A	99	to	103	>2
80	to	90	0.81	55	to	58	0.41	Settlement (ft)			0.2	Settlement (ft)			0.1
90	to	94	>2	58	to	62	0.31	Lateral Spreading (ft)			1.8	Lateral Spreading (ft)			0.3
94	to	100	>2	62	to	68	0.33								
100	to	105	>2	68	to	76	>2								
105	to	111	>2	76	to	83	>2								
Settlement (ft)			0.4	83	to	92	>2								
Lateral Spreading (ft)			4.2	92	to	98.1	>2								
				Settlement (ft)			0.6								
				Lateral Spreading (ft)			6.4								

Table B-11. Liquefaction Analysis Results for Project Boreholes

B-1				B-2				B-4			
Depth (ft)			FS	Depth (ft)			FS	Depth (ft)			FS
0	to	13	#N/A	0	to	18	#N/A	0	to	10.5	0.00
13	to	28.5	#N/A	18	to	42	#N/A	10.5	to	60	#N/A
29	to	57	#N/A	42	to	69	#N/A	60	to	76	#N/A
57	to	78.5	#N/A	69	to	81	#N/A	76	to	80	0.28
79	to	82	#N/A	Settlement (ft)			0.0	Settlement (ft)			0.3
Settlement (ft)			0.0	Lateral Spreading (ft)			0.0	Lateral Spreading (ft)			4.1
Lateral Spreading (ft)			0.0								

B-5				B-6				B-7			
Depth (ft)			FS	Depth (ft)			FS	Depth (ft)			FS
0	to	25	#N/A	0	to	18	#N/A	0	to	18	#N/A
25	to	47	#N/A	18	to	58	#N/A	18	to	42	#N/A
47	to	66	#N/A	58	to	78	#N/A	42	to	82	#N/A
66	to	78	#N/A	78	to	81.5	#N/A	Settlement (ft)			0.0
64	to	78	#N/A	Settlement (ft)			0.0	Lateral Spreading (ft)			0.0
78	to	81.5	0.38	Lateral Spreading (ft)			0.0				
Settlement (ft)			0.2								
Lateral Spreading (ft)			2.0								

B.7.4 Settlement and Consolidation of Foundation Materials

A consolidation analysis was conducted using one-dimension consolidation test results from five samples obtained from Marine Unit 1 (lean clay) occurring between approximate depths of 27 feet and 64 feet. All settlement and consolidation calculations herein are first-order approximations. For the stress range anticipated under project loading conditions (approximately 1 to 5 ksf) at the mid-point of the Marine Unit 1 layer, the calculated compressibility index (Cc) had a range of values as shown in Table B-2. The Boussinesq equation was used to determine the stress state change at the midpoint of the clay layer due to the load of the breakwater (including the pore pressure within the newly placed breakwater), and primary consolidation was determined as such:

$$\Delta p = \frac{qB}{B+z} - u$$

$$\Delta p = Stress\ change\ at\ midpoint\ of\ lean\ clay\ layer\ due\ to\ breakwater\ (psf)$$

$$q = Breakwater\ surcharge\ pressure\ (psf)$$

$$B = Width\ of\ breakwater\ footprint\ (ft)$$

$$z = Depth\ below\ mudline\ of\ lean\ clay\ layer\ midpoint\ (ft)$$

$$u = Pore\ pressure\ within\ the\ breakwater\ (ft)$$

$$\Delta p = \frac{(5029\ psf) \cdot (190\ ft)}{(190\ ft + 33\ ft)} - (19ft \cdot 64pcf) = 3290\ psf$$

$$S_c = \frac{C_c}{1+e_0} H_0 \log_{10}\left(\frac{p_0+\Delta p}{p_0}\right)$$ (Equation 7-7 from EM 1100-1-1905)

$$S_c = Consolidation\ settlement\ (ft)$$

$$C_c = Compressibility\ index\ (-)$$

$$e_0 = Initial\ void\ ratio\ (-)$$

$$H_0 = Thickness\ of\ clay\ layer\ (ft)$$

$$p_0 = Initial\ stress\ state\ at\ midpoint\ of\ clay\ layer$$

$$S_c = \frac{(0.45)}{1+(0.94)}(46\ ft)\log_{10}\left(\frac{(1454\ psf)+(3290\ psf)}{(1454\ psf)}\right) = 5.5\ ft$$

Assuming the mid-point of this layer is approximately 83 feet beneath the crest of the breakwater, the estimated primary consolidation settlement from this stratum is approximately 5.5 feet (based on previously established consolidation parameters). There is uncertainty in this approximation due to spatial variability of this layer, and consolidation settlement may range from approximately 3.2 feet to 8.9 feet, depending on the thickness and compressibility of the Marine Soils Unit 1.

Time rate of consolidation was also considered with material parameters derived from three samples taken from the Marine Unit 1 layer. A drainage length, H_{dr}, was for this layer was assumed based on one-way drainage as excess pore-pressure can only be relieved by draining to the mudline. The time to achieve 90% of consolidation, t_{90}, without ground improvement was calculated as:

$$T_{90} = Consolidation\ Time\ Factor\ (-)$$

$$H_{dr} = Drainage\ Length\ (ft)$$

$$c_v = Coefficient\ of\ Consolidation\ \left(\frac{ft^2}{s}\right)$$

$$q = Breakwater\ surcharge\ pressure\ (psf)$$

$$t_{90} = \frac{T_{90}H_{dr}^2}{c_v} \text{ (Equation 7-42 from EM 1100-1-1905)}$$

$$t_{90} = \frac{(0.84)(46ft)^2}{10^{-6}\frac{ft^2}{s}} = 1.78 \cdot 10^9 seconds = 20{,}572\ days$$

For the case where wick drains are installed on a 7-foot triangular grid, the H_{dr} reduces to 5 feet, and the time to achieve 90% of consolidation is reduced to 243 days.

These results are preliminary and are based on consolidation test data from the project site and the nearby 2015 Deep Water Dock Geotechnical data. These hand-calculations for magnitude and time-rate of consolidation should be confirmed with a Settle3 settlement/consolidation analysis during the PED phase. The PED settlement analysis should be conducted from crest elevation to a minimum depth of 150ft below mudline.

B.8 Geotechnical Engineering Evaluation

Preliminary evaluations indicate that the project is feasible provided ground improvement is incorporated to improve the performance of the underlying clay and that design adjustments are made to accommodate settlement.

Bearing capacity analyses show that the beach sand surface deposits are capable of supporting the breakwater with a factor of safety of 3.1. However, the underlying lean clay layer is a critical concern. In an undrained condition, its factor of safety is only 0.26, confirming it cannot support the structure without ground improvement. With wick drains assumed, the clay can dissipate excess pore pressures quicker and the factor of safety improves to 3.7, making this a viable mitigation measure. Settlement analyses further indicate that the clay could undergo 3.2–8.9 feet of primary consolidation.

To maintain crest elevation and freeboard, breakwater material quantities will need to be adjusted accordingly. Staged construction is recommended to allow for 90% of consolidation to occur before the placement of successive stages. Wick drains are recommended to accelerate consolidation during construction and reduce the risk of long-term performance loss.

Global slope stability analyses, using soil parameters from the site-specific geotechnical investigation and nearby investigations, also emphasize the importance of ground improvement. Results showed very low long-term factors of safety in the undrained case (0.3–0.5) and acceptable stability in the drained case for both the seaward and harbor side slope (1.6 and 1.5, respectively). These results reflect both the necessity of wick drains and the improved performance that will come from them.

Seismic analyses showed mixed results. Post-earthquake long-term stability was acceptable, with factors of safety of 1.3-1.4. However, pseudo-static analyses produced factors of safety less than 1.0. This deviation from the prescribed factor of safety is justified based on a risk-informed assessment of the breakwater's performance and failure consequences. The breakwaters failure would not pose a risk to human life as this breakwater is not meant to function as a life-safety structure. To mitigate for this seismic instability would be prohibitively expensive given the lack of risk to human life. Liquefaction analyses identified zones with factors of safety less than 2.0 but predicted settlements (≤ 0.6 feet) and lateral spreads (≤ 6.4 feet) are relatively small and unlikely to compromise performance.

Overall, the project is technically feasible with the inclusion of wick drains, settlement allowances in the design, and an exception to engineering guidance for seismic stability.

B.9 Future Investigations

Due to the wide range of possible settlement values, it is recommended that additional geotechnical information be collected in PED to characterize the spatial variability of consolidation along the breakwater alignment. It is recommended that, during PED, eight CPTU tests be conducted along the breakwater alignment to a minimum of 80 feet below mudline.

B.10 References

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ATTACHMENT A

2026 HOMER HARBOR FEASIBILITY STUDY GEOTECHNICAL DATA REPORT

ATTACHMENT B

2017 HOMER DEEP WATER DOCK GEOTECHNICAL INVESTIGATION REPORT

ATTACHMENT C

SEISMIC DESIGN PARAMETERS

ATTACHMENT D

SLOPE STABILITY RESULTS

ATTACHMENT E

LIQUEFACTION ANALYSIS

ATTACHMENT F

HOMER HARBOR GEOPHYSICAL REPORT